Performance of a damage-protected highway bridge pier subjected to bi-directional earthquake attack

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ABSTRACT: Recent earthquakes have demonstrated a need for a new design philosophy that avoids damage in order to ensure immediate functionality and reduced economic loss after an earthquake. Damage Avoidance Design (DAD) is one such philosophy that meets these demands. In this design approach, damage avoidance is obtained by special detailing of the joints, eliminating the formation of a plastic hinge and dissipating energy by rocking and supplemental dissipation devices. Quasi-static and pseudodynamic tests are performed on a scaled specimen to investigate the seismic behaviour of concrete bridge piers designed for damage avoidance. Input earthquake records are selected based on 50 and 90 percent survival probability as determined from an Incremental Dynamic Analysis (IDA). A 30% scaled circular pier designed to rock on a square steel-steel armoured interface is subjected to bi-directional lateral loading and axial load. Damage limit states are defined and confirmed by physical testing, which also verifies the bi-linear elastic, self-centring characteristics of the rocking DAD system.

1 INTRODUCTION

Current seismic engineering standards for reinforced concrete bridge piers tolerate a degree of inelastic behaviour when subjected to design level ground motions, resulting in the formation of a plastic hinge to provide ductile behaviour. As a direct consequence, damage to the pier is unavoidable, resulting in significant residual displacement causing closure of the bridge as the pier is repaired or replaced. As this methodology does afford economic (inelastic) structures with good life-safety (ductile) characteristics, damage is inherent and the economic costs of repair or replacement coupled with closure of transportation arteries can be devastating. As the end-user community is demanding more in terms of post-earthquake serviceability, a new *Damage Avoidance Design* (DAD) philosophy is emerging which will better resist damage and ensure serviceability demands are met.

The concept of rocking structures is an effective solution to this problem. Original investigations by Housner (1963) examined the free vibration behaviour of a rigid block. Subsequent studies considered flexibility (Meek 1978) coupled with rocking systems and prestress (Aslam et al. 1980) as a means of anchoring a structure to the ground and thus increasing its lateral capacity. More recently, these concepts have been carried over to bridge piers as presented by Mander and Cheng (1997) and Hewes and Priestley (2001); the philosophy was similarly brought to precast concrete beam-column "hybrid" joints and structural walls by Stanton et al. (1997) and Priestely et al. (1999). Though still not common, two state-of-the-practice examples can be found in New Zealand: the South Rangitikei Railway Bridge and an industrial chimney at Christchurch International Airport (Skinner et al. 1993).

Analytical and experimental investigations of such systems performed by Mander and Cheng (1997) adopt a *displacement-based design* (DBD) method to accurately determine force-deformation capacity through rigid-body kinematics. The method was confirmed by uni-directional cyclic loading and shake-table tests performed on reinforced concrete bridge piers, with and without un-bonded post-tensioning, and steel interface plates between the pier and foundation. No damage to the specimen was observed, however, bi-directional tests were not performed which would better represent actual ground motions.

Additional analytical methods (Pampanin 2001) were developed for precast concrete beam-column "hybrid" joints with un-bonded post-tensioning and energy dissipation devices using an "equivalent monolithic beam" analogy to analyse a section by an iterative procedure. Studies by Palermo (2004) have investigated the validity of the "hybrid" controlled rocking system when applied to bridge piers and the global response of this system with regular and irregular pier configurations.

This study will further investigate the seismic response of rocking bridge piers. As an extension to the uni-directional tests performed by Mander and Cheng (1997), a scaled bridge pier with an armoured interface is subjected to quasi-static and pseudodynamic bi-directional loading patterns. Special attention will be given to large concentrated forces which must be transmitted through a small region of the specimen due to bi-directional rocking behaviour. Damage will be classified according to an established indexing system and compared to that of a conventional ductile bridge pier.

2 EXPERIMENTAL PROGRAM

2.1 *Prototype Design*

The prototype bridge pier is 7m high and taken from a typical 'long' multi-span highway bridge on firm soil with 40m longitudinal spans and a 10m transverse width. Design details are presented in Figure 1. The seismic weight of the superstructure was calculated to be 7000kN. The pier was assumed to be located in a high seismic zone in New Zealand, with the DBE being 0.4g. The moment demand was assessed according to the New Zealand seismic design standard (NZS:3101 1995) considering a ductile monolithic pier; this was calculated to be 7436kN-m.

The moment capacity, M, of the DAD pier is provided by a combination of gravity load, longitudinal un-bonded tendons, and supplemental energy dissipation devices. Assuming the pier will behave essentially rigid, rigid body kinematics can be employed to predict the pier's response to seismic excitation. The moment necessary for uplift, deemed M_y, can be calculated by:

$$M_y = (P+F)\frac{B}{2} + A_s\sigma_y\frac{2e+B}{2} \qquad (1)$$

where P = axial load from gravity; F = effective prestress force; B = width of the rocking base; A_s = cross sectional area of the energy dissipaters; σ_y = yield stress of energy dissipaters; e = eccentricity of the energy dissipaters from centreline. Since P is in effect fixed, the required moment capacity can be reached by modifying the geometry of the rocking interface, adding additional prestress, or adding dissipaters. The post-yield behaviour of the pier is a function of the initial prestress and the elastic properties of the tendons.

To determine the displacement of the pier at uplift, it is necessary to consider the elastic behaviour of the pier prior to this point. This is a function primarily of the moment of inertia, I. Common practice would consider an effective moment of inertia, I_{eff}, from a crude function of I_{gross}, which is calculated from the gross geometry of the section. I_{gross} is essentially constant throughout the length of the pier, however, once rocking begins to occur and the axial loads are transmitted to a smaller portion of the base, I_{gross} begins to reduce at a rapid rate. This is due to the St. Venant principle, where a portion of the pier is rendered ineffective, cut 45° from the point of rocking. Therefore, it can be shown that I_{eff} can be taken as $0.25I_{gross}$. This effective stiffness is used in subsequent analytical predictions.

Since the post-yield response of the DAD pier is limited to the rocking region, it is implied that the pier itself will not form a plastic hinge, and can therefore be detailed according to nominal longitudinal and transverse reinforcement requirements.

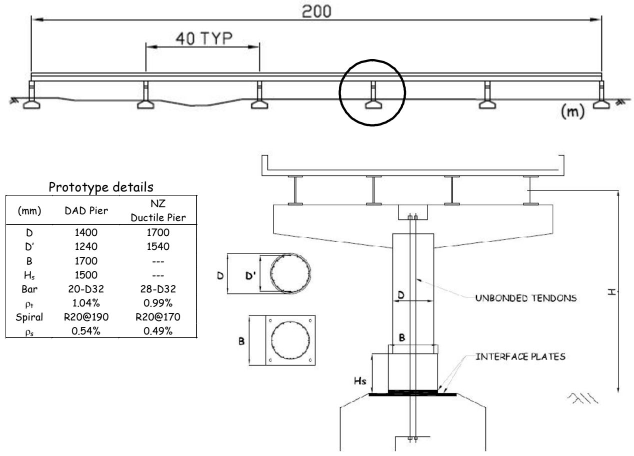


Figure 1. Design details of the prototype bridge pier.

2.2 *Specimen Construction*

A 30% scaled model of the prototype bridge pier was constructed. It was scaled to keep the same longitudinal and transverse reinforcement ratios as in the prototype pier. The specimen was constructed in four parts: (i) the base block; (ii) circular column; (iii) head block; and (iv) shoe block. Figure 2 gives the design details at the armoured interface and Figure 3 illustrates the specimen in the testing apparatus. The base block and head block were considered part of the experimental testing apparatus, and therefore designed to withstand the expected demands from testing. Plate C, with a 350x350 mm hole, was placed flush with the top of the base block to act as the armoured rocking interface of the pier's foundation. To construct the interface at the base of the column, plate B was bolted to plate A to form the shear key which would rest in the square hole of plate C. A 3mm gap was provided on each side to prevent the steel plates from grinding during rocking. Longitudinal reinforcement was tack welded into holes drilled in plate A. The R6 spirals were wrapped around these longitudinal bars. The pier was poured without completing the shoe block. To finish the shoe block, a portion of the cover concrete of the pier was jack hammered off to expose the longitudinal reinforcement. Three D16 grade 500

MPa bars were tack welded to plate A at each corner and to the pier's longitudinal reinforcement, creating a diagonal mechanism to resist expected strut forces. Additional D16 'hoop' bars were placed parallel to each plate edge. Two layers of high strength wire rope (7x19 construction) were wrapped around the inner diagonal reinforcement and the outer cage. This is expected to better confine the concrete and prevent excessive cracking. The shoe block was poured separately from the circular pier using a high strength concrete mix with 1% (by weight) DRAMIX fibres. Figure 4a presents a photo of the reinforced shoe block.

The energy dissipaters consisted of R12 threaded bars with the central 150mm segment machined to 7mm diameter. These devices were screwed vertically into plate C through ducts at each corner of the shoe block, bolted in place, and stressed to $0.5f_y$ by a torque wrench. Since damage from earthquakes can largely be attributed to large initial pulses, particularly during near-field events, these dissipaters are designed to perform in tension only, with the intent they could be easily replaced following a seismic event.

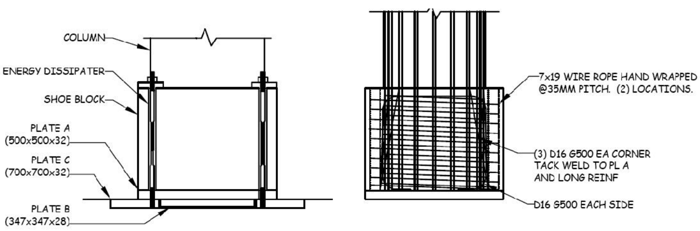


Figure 2. Design details of the DAD specimen shoe block.

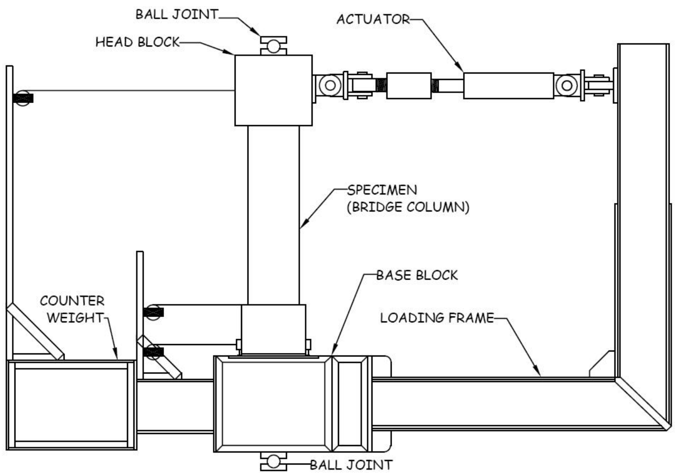
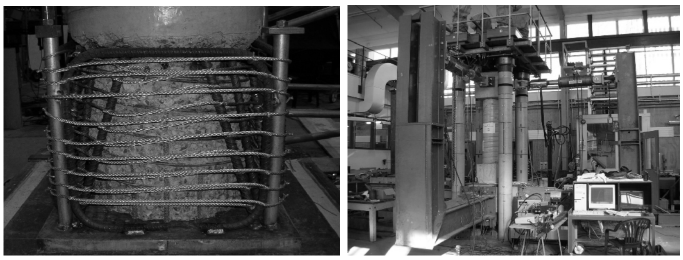


Figure 3. East-west elevation of the testing apparatus. The layout is identical in the north-south direction.

2.3 *The Test Apparatus*

The testing apparatus (Fig. 3) was designed to simulate actual seismic demands imposed on the prototype structure. To accomplish this, simultaneous lateral loads combined with axial load were applied to the specimen via two actuators mounted on reaction frames. These frames were assembled within the confines of a 10,000 kN capacity DARTEC universal testing machine. At top and bottom, a ball joint transmitted a constant axial load of 777 kN, consisting of the weight of the superstructure (630 kN) and the simulated force from un-bonded tendons (147 kN). The L-shaped reaction frames were attached to counter weight baskets by 30mm diameter high strength rods running through the base block. Lateral loads were applied via 800 kN capacity hydraulic actuators, connected to the specimen's head block and reaction frame by universal joints.

A primary rotary potentiometer was installed in line with each actuator; these measured the displacement of the specimen for use by the controller's pseudodynamic algorithm. Two additional rotary potentiometers were installed at the top and bottom of the shoe block along with a series of spring potentiometers at its corners to measure localized uplift. All instrumentation was isolated on the testing apparatus to measure relative displacement. Load cells (1000 kN capacity) were installed in-series with the actuators. A photograph of the testing apparatus is given in Figure 4b.



(a) (b)

Figure 4. Photos prior to testing: (a) the reinforced shoe block; (b) the testing apparatus.

2.4 *Testing Program*

Testing consisted of a series of quasi-static and pseudodynamic tests. Results from pseudodynamic testing only will be presented here. The concept of pseudodynamic testing basically consists of using a physical model of a structure to determine its stiffness in real time and thus calculate displacement of the structure when subject to real earthquake (acceleration) records. This is accomplished by solving the fundamental equation of motion. A good overview of this concept is given in Shing et al. (1996).

To determine the earthquake records used for pseudodynamic testing a procedure described by Dhakal et al. (2006) was adopted. In their study, pseudodynamic testing was performed on a bridge pier designed to the New Zealand standard. Earthquake records were selected based on *Incremental Dynamic Analysis* (IDA); an inelastic analytical model was subject to a suite of earthquakes over a range of scaled intensities. A suite of twenty earthquakes were adopted as potential candidates based on a study conducted by Vamvatsikos and Cornell (2004). These records range in magnitude between 6.5 and 6.9, have moderate epi-central distance, and were recorded on firm soil. The IDA data from these records was analysed probabilistically to identify those critical to the piers. Records were chosen to represent the 90th percentile *design basis earthquake* (DBE), with a 10% chance of occurrence in 50 years, and the 50th and 90th percentile *maximum considered event* (MCE), with a 2% chance of occurrence in 50 years. Since the present study will highlight the enhanced performance of a DAD bridge pier, it is necessary to directly compare its performance to that of a conventional ductile pier. To accomplish this, specific earthquake selection for the DAD pier was not performed; the same earthquakes selected for the New Zealand ductile pier have been adopted for this study. These earthquakes, termed EQ1 (90% DBE), EQ2 (50% MCE), and EQ3 (90% MCE), are given in Table 1. For testing, EQ1, EQ2, and EQ3 were applied consecutively, with 5 second intervals of zero acceleration between each record. This interval allowed the residual drift and natural period of the structure to be recorded.

Table 1. Earthquake records adopted for pseudodynamic testing.

	Comp.	PGA(g)	Event	Year	Station
EQ1	EW NS	0.376 0.400	Imperial Valley	1979	Chihuahua
EQ2	EW NS	0.800 0.787	Loma Prieta	1989	Anderson Dam
EQ3	EW NS	0.800 0.700	Superstition Hills	1987	Wildlife Liquefaction

2.5 *Damage Classification*

This study adopts the five *damage states* (DS1 to DS5) defined by Mander & Basoz (1999) that have been used in *Hazus*, as summarized in Table 2. Observed damage to the pier is classified according to this index.

Table 2. Damage states index according to *Hazus*.

Damage State		Repair Required	Outage
DS1	None	None	None
DS2	Minor	Inspect, Adjust, Patch	< 3 days
DS3	Moderate	Repair Components	< 3 weeks
DS4	Major	Rebuild Components	< 3 months
DS5	Complete	Rebuild Structure	> 3 months

3 TEST RESULTS

The specimen was subject to two pseudodynamic tests: the first without energy dissipaters and the second with energy dissipaters. Figure 5 presents results from testing with energy dissipaters. Similar to the CLT test, the results are plotted so that data from one graph is projected to the next, resulting in two force-displacement curves (Figure 5a & b), two displacement history curves (Figure 5d & e) and a plan view of bi-directional displacement (Figure 5c). The maximum drift recorded for EQ1 in the EW and NS direction was 1.91% and -1.80% at 6.48 seconds and 14.94 seconds, respectively. The corresponding lateral forces were 90.58kN and -80.43kN. There was no damage to the pier from EQ1, aside from some minor hairline cracks diagonally from the bottom corners to top midsection of the shoe block. Additionally, tensile bending cracks were observed along the circular pier; these closed and were undetectable after testing. After EQ1, the pier was classified at DS1: no damage.

The maximum drift observed during EQ2 in the EW and NS direction was 3.83% and 2.15% at 36.42 seconds and 38.34 seconds, respectively. The corresponding lateral loads were 96.45kN and 90.85kN. A photograph of the shoe block at approximately 3% drift (at 37 seconds) is given in Figure 6a. At times the pier was rocking on a single corner of the shoe block, such as when the drift of the pier was at 2% in both the NS and EW direction at approximately 39 seconds. This resulted in minor crushing and additional hairline cracks propagating from the shoe block corners to the pier, as shown in Figure 6b. Such damage was largely aesthetic and did not cause noticeable degradation of strength or stiffness. However, since it may require repair, the pier was classified as being at DS2.

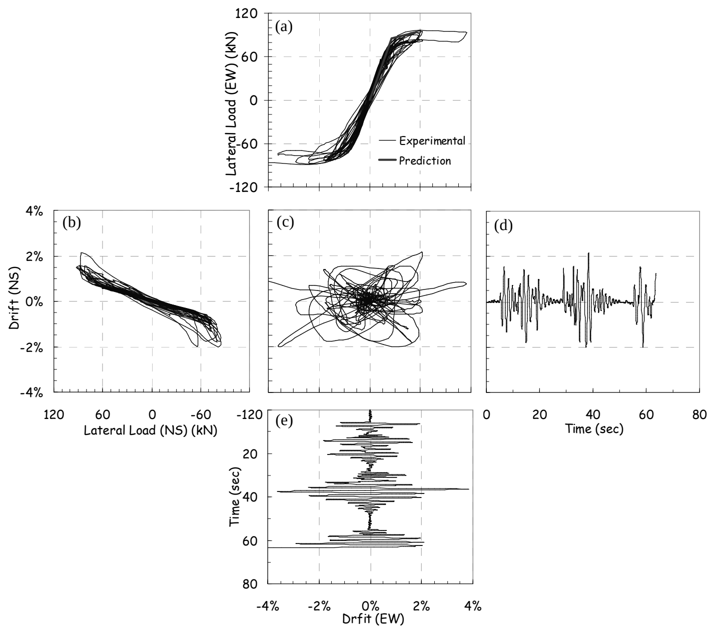
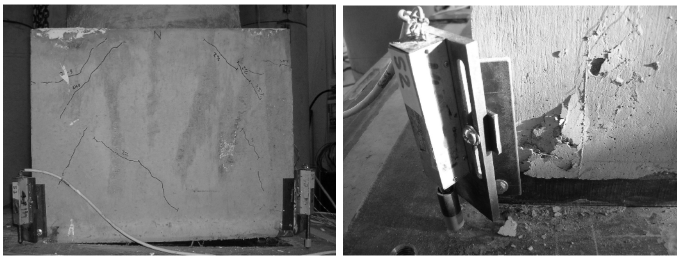


Figure 5. Test results of the DAD pier with energy dissipaters, subjected to EQ1, EQ2, and EQ3: force-displacement curves for (a) EW with the analytical prediction pushover curve and (b) NS direction; (c) plan view of drift; (d) EW and (e) NS displacement-time plot.

The pier did not survive EQ3. At 13.5 seconds the displacement of the pier was 5.5% in the EW direction and 1.2% in the NS. It was deemed unsafe to continue testing, thus resulting in an assumed complete collapse of the structure under EQ3. Aside from this, there appeared to be only minor additional damage. Further crushing and hairline cracking was observed, but not enough to justify damage beyond DS2. However, since complete collapse was assumed due to termination of the test, the pier was classified at DS5.



(a) (b)

Figure 6. Photos during testing: (a) the shoe block rocking at 3% drift; (b) localized crushing at the shoe block corner.

Figure 7 presents a comparison of the hysteretic and displacement history response of the pier with and without energy dissipaters. Results are presented for EQ2. The energy dissipation devices had a small contribution to the overall behaviour of the specimen. The maximum lateral force in the EW and NS direction for EQ1 increased 3% and 4%, respectively. For EQ2 there was a similar increase in strength, 7% and 4% for the EW and NZ, respectively.

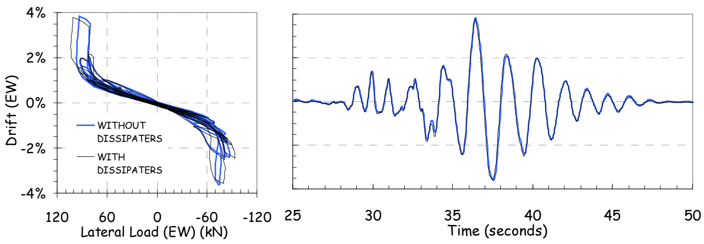


Figure 7. Comparative plot of testing with and without energy dissipaters during EQ2.

Strain gauges attached to the longitudinal steel 50mm from the base did not detect yielding, which would have occurred at approximately 1% drift had the specimen been a conventional monolithic pier. Supplemental instrumentation recorded that rocking of the shoe block was generally about 1% drift less than rocking of the entire specimen. Therefore, the pier itself did not exceed 1% drift throughout all phases of testing. During the zero acceleration portion of testing no measurable residual displacement was observed.

4 COMPARISON WITH A CONVENTIONAL DUCTILE PIER

To highlight the advantages of DAD, the specimen's performance was compared to that of conventional monolithic pier (Dhakal et al. 2006) designed to the seismic design codes of New Zealand (NZS:3101 1995). The prototype details are given in Figure 1. The pier was subject to the same pseudodynamic testing as the DAD pier. Figure 8 presents the hysteretic behaviour and displacement profile of the two piers for EQ1 and EQ2. It is apparent from this figure that the ductile pier has a higher stiffness and moment capacity than the DAD pier. This is due to the different design procedures of the two piers. Consequently, maximum displacement for the two piers varied considerably. However, the DAD pier suffered considerably less damage and residual displacement. After EQ1, the NZ pier was classified as being at DS2, and stayed at DS2 even after EQ2. The residual drift for the DAD pier was virtually zero; for the ductile pier it was approximately 0.25%. After EQ3, testing was terminated due to high drifts, resulting in DS5. Damage was comparably less to the DAD pier. Although the final collapse condition was similar for EQ3, considerably less damage was observed after EQ1 and EQ2.

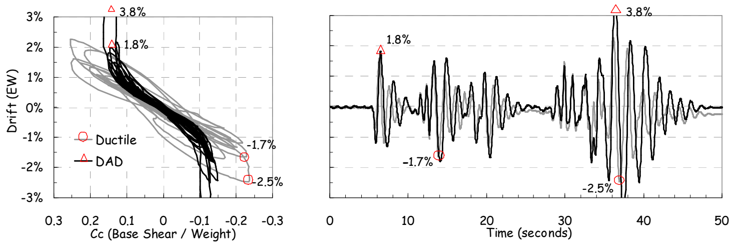


Figure 8. Experimental comparison between a conventional ductile pier and the DAD pier subjected to EQ1 and EQ2.

5 DISCUSSION

This research further investigates the application of DAD. A 40m span prototype bridge was designed using discontinuous longitudinal reinforcement at the column-foundation interface to allow rocking at a specially detailed armoured joint. Lateral forces were resisted by gravity load, post-tensioned tendons (simulated during testing) and supplemental energy dissipation devices. A 30% scaled model was constructed and tested with pseudodynamic bi-directional lateral forces and axial load. Results confirmed bi-linear elastic behaviour with negligible residual displacement. Some energy dissipation was observed even when dissipaters were not being used. This was likely caused by friction within the testing apparatus, particularly at the ball joints. Damage to the pier was minor until a drift of approximately 5.5%. Toppling was assumed to occur when testing was terminated during EQ3 (90% MCE) due to safety considerations.

Special attention was given to the resistance of large concentrated compressive forces resulting from bi-directional rocking at an extreme corner of the steel-steel interface. Even at high drifts, only minor damage from superficial crushing and hairline cracks was observed. This can be attributed to the diagonal reinforcing bars which transferred the strut forces into the pier and the fibre reinforcing which impeded crack propagation. The energy dissipation devices did not significantly contribute to the pier's performance. Further development of these devices is needed to provide more efficient, reliable energy dissipation.

Based on the probabilistic nature of the earthquake selection process, it is possible to state the likely outcomes of damage in a *performance-based earthquake engineering* (PBEE) context. For example, it can be stated that there is some 90% confidence the DAD pier will survive an earthquake that has a 10% chance of occurrence in 50 years (DBE) with no damage. For an earthquake that has a 2% chance of occurrence in 50 years (MCE), there is 50% confidence that the structure will only sustain minor damage, yet it cannot be said the structure will survive with some 90% confidence. Similarly for the ductile pier, it can be stated that it has a 90% probability of surviving a DBE without exceeding DS2.

There are several obvious benefits of DAD apparent from this study: (i) a lack of damage can potentially lead to lower operating and repair costs; (ii) negligible residual displacement will ensure serviceability following a seismic event; (c) pre-cast construction can be utilized to increase reliability and reduce initial (construction) costs.

6 CONCLUSIONS

Based on physical testing of a DAD bridge pier, the following conclusions can be drawn:

– With 90% confidence, it can be stated that the DAD pier will not sustain damage from a design basis earthquake. For a maximum considered earthquake, the structure may have minor damage and there is at least a 50% confidence the pier will not collapse.
– Concentrated axial loads were resisted by special detailing at the column-foundation interface. This concentration was resisted by a combination of reinforcing steel and high strength fibre-reinforced concrete. Only minor damage was observed under bi-directional loading up to 5.5% drift in the pier.
– The energy dissipation devices utilized in this study provided some additional lateral resistance. These or similar devices are recommended, though more efficient designs may increase their contribution to lateral resistance and dissipation of earthquake energy.
– No stiffness degradation or residual displacement was observed. This was shown to be due to the rocking, bi-linear elastic hysteretic behaviour of the pier.

ACKNOWLEDGEMENTS

Generous support from the sponsors of the *Future Building Systems* research project is greatly acknowledged, particularly the *Foundation for Research, Science and Technology* (FRST).

REFERENCES

Aslam, M. Goddon, W.G. and Scalise, D.T. 1980. Earthquake Rocking Response of Rigid Blocks. *Journal of Structural Engineering, ASCE*; 106(2): 377-392.

Dhakal, R.P. Mander, J.B. & Mashiko, N. 2006. Identification of critical ground motions for seismic performance assessment of structures. *Earthquake Engineering and Structural Dynamics*; (in press).

Hewes, J.T. and Priestley, M.J.N 2001. Experimental Testing of Unbonded Post-tensioned Precast Concrete Segmental Bridge Columns. *Proceedings of the 6th Caltrans Seismic Research Workshop Program*, Radisson Hotel, Sacramento, California, June 12-13, Div. of Engineering Services, California Dept. of Transportation, Sacramento, 8 pages.

Housner, G.W. 1963. The Behavior of Inverted Pendulum Structure During Earthquake. *Bulletin of the Seismological Society of America*; 53(2): 403-417.

Mander, J.B. and Cheng, C.T. 1997. Seismic Resistance of Bridge Piers Based on Damage Avoidance Design. *Technical Report NCEER-97-0014* (National Center for Earthquake Engineering Research). State University of New York, Buffalo, December 10 1997.

Mander, J.B. Basoz, N. 1999. Seismic fragility curve theory for highway bridges in transportation lifeline loss estimation. *Optimizing Post-Earthquake Lifeline System Reliability*, TCLEE Monograph No. 16. American Society of Civil Engineers: Reston, VA, U.S.A.; 31–40.

Meek, J.W. 1978. Dynamic Response of Tipping Core Buildings. *Earthquake Engineering & Structural Dynamics*; 6(5): 437-454.

NZS:3101-95. 1995. Concrete Structures Standard: NZS3101. *Standards New Zealand*. Wellington.

Palermo, A. Pampanin, S. Calvi, G.M. 2004. Use of 'Controlled Rocking' in the Seismic Design of Bridges. *Proceedings of the 13th World Conference on Earthquake Engineering*; August 1-6, Vancouver, B.C., Canada. Paper No. 4006.

Pampanin, S. Priestley, M.J.N Sritharan, S. 2001. Analytical Modelling of the Seismic Behavior of Precast Conrete Frames Designed with Ductile Connections. *Journal of Earthquake Engineering*; **5**(3): 329-367.

Park, R. Pauley, T. 1975. *Reinforced Concrete Structures*; John Wiley & Sons, Inc., New York, NY.

Priestley, M.J.N. Sritharan, S. Conley, J.R. Pampanin, S. 1999. Preliminary Results and Conclusions from the PRESSS Five-Story Precast Concrete Test Building. *PCI Journal*; 44(6): 42-67

Shing, P.B. Nakashima, M. Bursi, O.S. 1996. Application of dynamic test method to structural research. *Earthquake Spectra*; 12(1): 29-54

Skinner, R.I. Robinson, W.H and McVerry, G.H. 1993. *An Introduction to Seismic Isolation*. John Wiley & Sons, Inc.; New York, NY.

Stanton, J.F. Stone, W.C. Cheok, G.S. 1997. A Hybrid Reinforced Precast Frame for Seismic Regions. *PCI Journal*; 42(2): 20-32.

Vamvatsikos, D. and Cornell, C.A. 2004. Applied Incremental Dynamic Analysis. *Earthquake Spectra*; 38(2).